REVIEW OF MECHANICAL PROPERTIES OF HSC AT ELEVATED TEMPERATURE

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ABSTRACT: A compilation of experimental results on the mechanical properties of concrete when exposed to rapid heating as in a fire are presented. Emphasis is placed on concretes with high original compressive strengths, that is, high-strength concretes (HSCs). The compiled test data were categorized by the test methods and the types of aggregate used and compared for behavioral differences. The comparison revealed distinct difference in mechanical properties of HSC and normal-strength concrete (NSC) in the range between room temperature and approximately 450°C. The differences narrowed at temperature above 450°C. Also presented is a comparison of these test results with existing code provisions on the effects of elevated temperature on concrete strength. It is shown that the Eurocode provisions and the Comités Euro-International Du Beton (CEB) design curves are more applicable to NSC than to HSC. In fact, these provisions are unsafe when compared with HSC test results. The study showed a lack of experimental data for lightweight HSC and HSC heated under a constant preload to simulate the stress conditions in HSC columns.

INTRODUCTION

High-strength concrete (HSC) can now be manufactured by most concrete plants using state-of-the-art additives, such as silica fume and water-reducing admixtures. The HSC offers significant economic and architectural advantages over ordinary concrete and also is suited for special constructions that require high durability. These advantages, coupled with availability in many local markets, have resulted in increased use of HSC in a variety of applications.

However, results of many recent fire tests have shown that there are significant differences between the performance of HSC at elevated temperature compared with ordinary concrete. These include the differences in mechanical properties (compressive strength and elastic modulus) retained by HSC and normal-strength concrete (NSC) at elevated temperature and a higher potential of HSC specimens to fail by explosive spalling when subjected to rapid heating. This raises questions about the applicability of current design provisions for fire-exposed concrete such as those prescribed by the Eurocode and the Comités Euro-International Du Beton (CEB) (CEB 1991), which are based on the performance of ordinary concrete, to HSC structures. Furthermore, the explosive spalling failure mechanism has been observed on an inconsistent basis. Often, explosive spalling has occurred in only a few of a larger group of specimens subjected to identical testing conditions. This erratic behavior makes it difficult to predict with certainty whether a given HSC will fail by explosive spalling when exposed to fire. It has been theorized that the higher susceptibility of HSC to explosive spalling is caused by, in part, the lower permeability of HSC, which limits the ability of heated moisture to escape from within the concrete. This results in a buildup of pore pressure within the cement paste. As heating increases, the pore pressure also increases. This increase in vapor pressure continues until the internal stresses become so large as to cause explosive spalling of the heated concrete.

Given the potential benefits of HSC and its increased usage, questions about its performance at elevated temperature need to be examined. Also, the applicability of existing fire-design provisions needs to be evaluated and new provisions established where necessary. It should be noted that the differentiation between HSC and NSC in this paper follows American Concrete Institute (ACI) 393R-92's definition (ACI 1992), which considers concretes with design compressive strength of 40 MPa or greater as HSC. Although this is convenient for a comparison purpose, it should be mentioned that, by today's standards, 40-MPa concrete is too low to be considered typically as HSC.

OBJECTIVES

This paper provides (1) a systematic comparison of results of high-temperature tests on NSC and HSC specimens, conducted by various researchers, to examine the effect of high-temperature exposure to the mechanical properties of concretes with different original compressive strengths; (2) an examination of the applicability of current design recommendations for mechanical properties of HSC at elevated temperature; and (3) a general discussion on the research needs for fire performance of HSC.

REVIEW OF EXPERIMENTAL STUDIES ON FIRE-EXPOSED HSC

The effects of high temperature on the mechanical properties of concrete have been investigated as early as the 1940s, and it has been established that the mechanical properties of concrete are modified with high-temperature exposure. These early studies used primarily NSC specimens and some subjected the specimens to long-term, sustained heating. Only in more recent studies, which include tests by Abrams (1971); Hertz (1984); Diederichs et al. (1988); Castillo and Durani (1990); Morita et al. (1992); Sullivan and Shansar (1992); Furumura et al. (1995); Hammer (1995); Felicetti et al. (1996); and Noumowe et al. (1996), were HSC specimens used in high-heating rate tests. Because the focus of this paper is on the effect of high-temperature exposure, as in the case of fire on properties of HSC, only results of the foregoing test programs are compiled here. Detailed summaries of these studies may be found in Phan (1996). Key features are summarized in Table 1.

The specimens used in these test programs consisted of both concrete prisms and cylinders of various sizes (100 × 100 × 100 mm to 80 × 275 × 500 mm for concrete prisms, and 28 × 52 mm to 160 × 320 mm for concrete cylinders). The specimens were made using a combination of concrete mixtures. Some were conventional portland cement; others in-

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TABLE 1. Summary of Materials Test Programs Reviewed

Test programs (1)	Compressive strength (MPa) (2)	Test Methods Used I (3)	II (4)	III (5)	Concrete or aggregate type (6)	Specimen size (mm) (7)	Observed explosive spalling (8)
Castillo and Durani (1990)	28, 62[a] 31, 63, 89[b]	X	X		Conventional portland cement	51 × 102[c]	320–360°C
Hertz (1984)	170[a] 150[b]			X	Silica fume concrete with steel fiber	100 × 200[c] 57 × 100[c] 28 × 52[c]	350–650°C
Diederichs et al. (1988)	33–114[a]		X		Blast furnance slag, silica fume, fly ash, OPC	100 × 100 × 100[d] 80 × 300[c]	350°C
Hammer (1995)	69–118[b]		X		Silica fume concrete with LWA and NWA	100 × 310[c]	300°C
Sullivan and Shansar (1992)	38–65[a]		X	X	Combinations of cement, silica fume, slag, and two types of aggregate	64 × 64[c]	Not observed
Abrams (1971)	23–45[a]	X	X	X	NWA (carbonate and siliceous) and LWA	75 × 150[c]	Not observed
Morita et al. (1992)	20, 39, 59[a] 20, 74[b]			X	Conventional portland cement	100 × 200[c]	Not observed
Furumura et al. (1995)	21, 42, 60[a] 38, 55, 79[b]		X	X	Conventional portland cement	50 × 100[c]	300°C
Felicetti et al. (1996)	72, 95[a]			X	Silica fume concrete	100 × 300[c] 100 × 150[c] 80 × 275 × 500[d]	Not observed
Noumowe et al. (1996)	38, 61			X	Silica fume with calcareous aggregate	160 × 320[c] 100 × 100 × 400[d]	300°C

Note: I, stressed test method; II, unstressed test method; III, unstressed residual–strength test method.
[a]Designed concrete strength.
[b]Measured concrete strength at time of fire testing.
[c]Specimens are cylinders, the first dimension is the diameter and the second is the height.
[d]Prism specimens.

cluded additives such as silica fume, fly ash, and steel fibers. The type of coarse aggregate used included normal-weight (NWA) calcareous, siliceous, and lightweight aggregates (LWA). The specimens' compressive strength at testing ranged from 20 to 150 MPa. The studies employed three variations of the steady-state temperature test methods, which are commonly referred to as stressed, unstressed, and unstressed residual–strength tests. Each of these test methods yields results that are typical of the internal stress conditions of specific concrete structural elements. For example, the results of the stressed test are most suitable for representing fire performance of concrete in a column or in the compression zone of a beam, because in this test, a preload, often in the range of 20–40% of the ultimate compressive strength at room temperature, is applied to the concrete specimen prior to heating and sustained during the heating period. Heat is applied at a constant rate until a target temperature is reached and maintained until a thermal steady state is achieved. Load or strain is then increased at a prescribed rate until the specimen fails. The results of the unstressed test are most suitable for representing fire performance of concrete in the tension zone of a beam or concrete in an element that has a small compressive load, because in this test, the specimen is heated, without preload, at a constant rate to the target temperature and maintained until a thermal steady state is achieved. Load or strain is then applied at a prescribed rate until failure occurs. And finally, the results of the unstressed residual–strength test are suitable for use in assessing the postfire (or residual) properties of concrete, because in this test the specimen is heated without preload at a prescribed rate to the target temperature and maintained until a thermal steady state is achieved. The specimen is then allowed to cool, also following a prescribed rate, to room temperature. Load or strain is applied at room temperature until the specimen fails.

As summarized in Table 1, explosive spalling was observed in some but not all of these test programs. The lowest temperature at which explosive spalling occurred was reported to be about 300°C (Hammer 1995) and the highest was about 650°C. Also, it should be noted that within the same test pro-

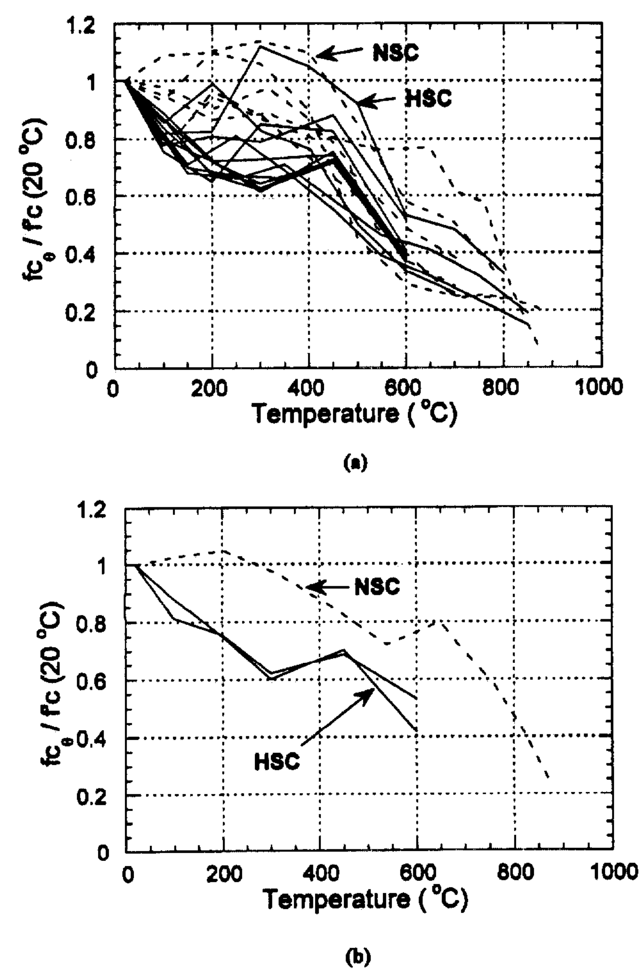


FIG. 1. Compressive Strength–Temperature Relationships for: (a) NWA Concretes; (b) LWA Concretes as Obtained by Unstressed Tests

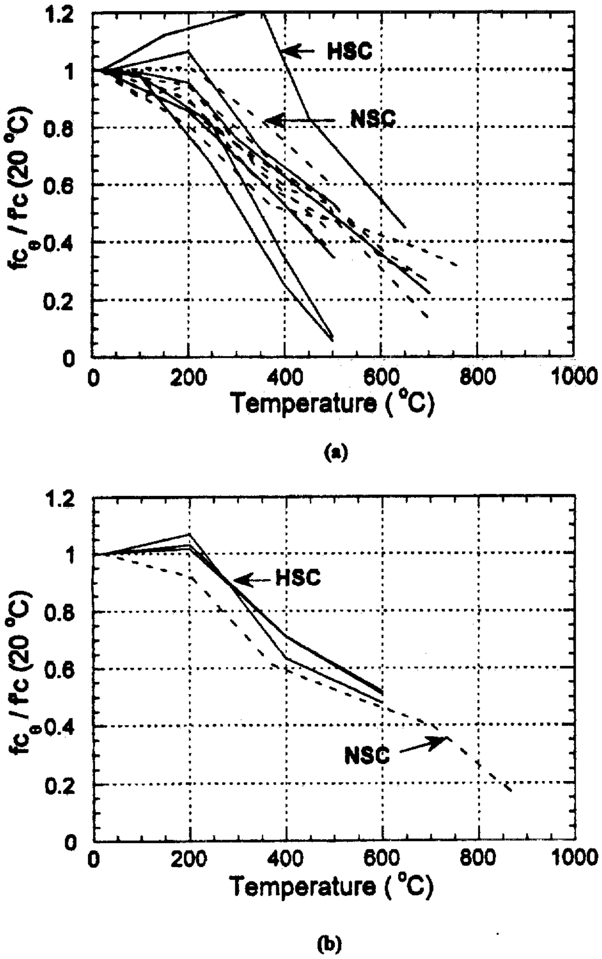


FIG. 2. Compressive Strength–Temperature Relationships for: (a) NWA Concretes; (b) LWA Concretes as Obtained by Unstressed Residual-Strength Tests

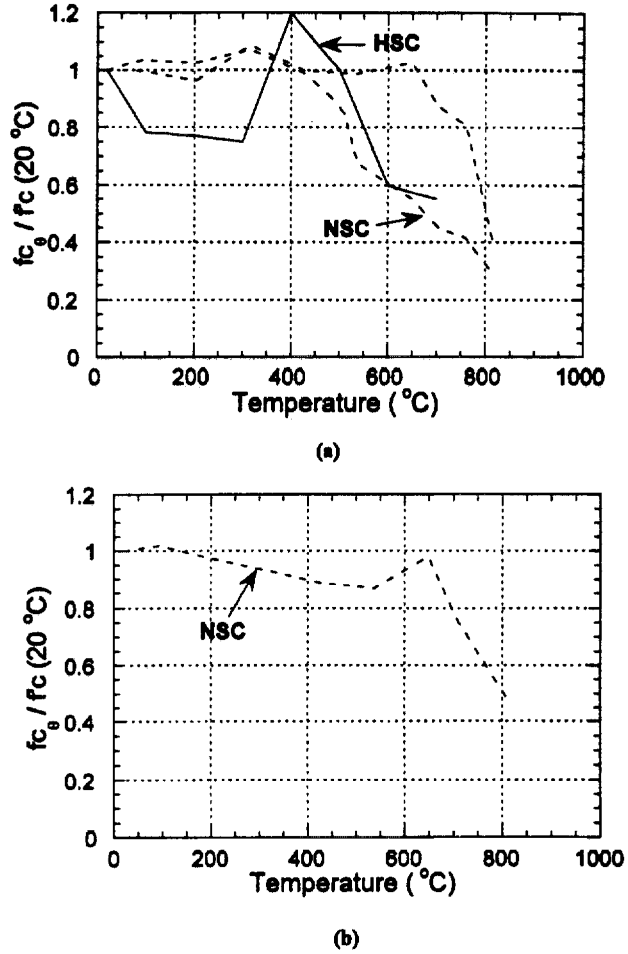


FIG. 3. Compressive Strength–Temperature Relationships for: (a) NWA Concretes; (b) LWA Concretes as Obtained by Stressed Tests

grams, explosive spalling was not always observed in replicate specimens. Despite this inconsistency, it is believed that explosive spalling occurs under certain combinations of test conditions and that higher-strength concretes, especially those densified with silica fume, are more susceptible to this type of event.

The results of the foregoing test programs are compiled and shown in the forms of compressive strength and elastic modulus versus temperature relationships in Figs. 1–5. These relationships are distinguished by the test methods used, i.e. unstressed, unstressed residual strength, and stressed tests, and by the aggregate types (NWA or LWA). The relationships for HSC (greater than 40 MPa) are shown by solid lines, and relationships for NSC (less than 40 MPa) are shown by dashed lines. The effects of other parameters such as specimen size, specimen shape, heating rate, cement content, proportion of additives, etc. are not distinguished in the analyses.

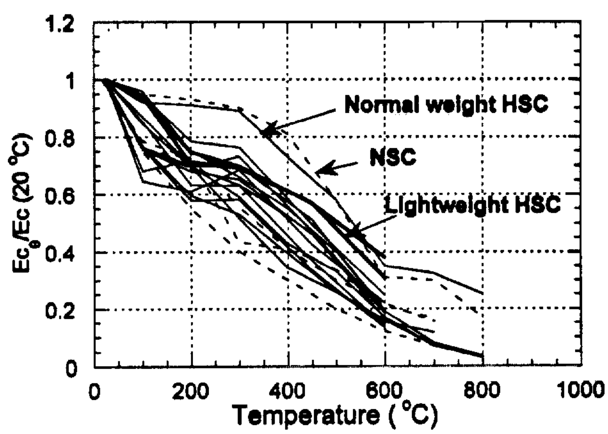


FIG. 4. Modulus of Elasticity–Temperature Relationships for NWA and LWA Concretes from Unstressed Test Results

Effect of Temperature on Compressive Strength of HSC

The results of the unstressed tests are compiled in Figs. 1(a and b) for NWA and LWA concretes, respectively. As shown in these figures, the compressive strength–temperature relationships of the unstressed tests are characterized by the following three stages: (1) Initial strength–loss stage, which begins from room temperature to anywhere between 100 and 200°C for NWA concrete and from room temperature to 250°C for LWA concrete; (2) stabilizing and regaining stage, which begins from anywhere between 100 and 200°C to anywhere between 400 and 450°C for NWA concrete and from between 250 and 450°C for LWA concrete; and (3) permanent strength–loss stage, which begins anywhere between 400 and 450°C for NWA concrete and from anywhere between 250 and 450°C for LWA concrete. The unstressed strength–temperature relationships for HSC appear to follow similar trends as for NSC, except that the loss of strength in temperature range between 25 and approximately 400°C for HSC is noticeably greater than the loss of strength for NSC. This difference is narrowed in the permanent strength–loss stage.

The unstressed residual–strength test results are compiled in Figs. 2(a and b) for NWA and LWA concretes, respectively.

The compressive strength–temperature relationships of HSC are characterized by the following two stages: (1) Initial strength gain or minor strength–loss stage, which begins from room temperature to approximately 200°C for both NWA and LWA concretes; and (2) permanent strength–loss stage, which begins from approximately 200°C for both NWA and LWA concretes. The unstressed residual strength–temperature relationships for HSC and NSC are somewhat similar for the entire range of temperature.

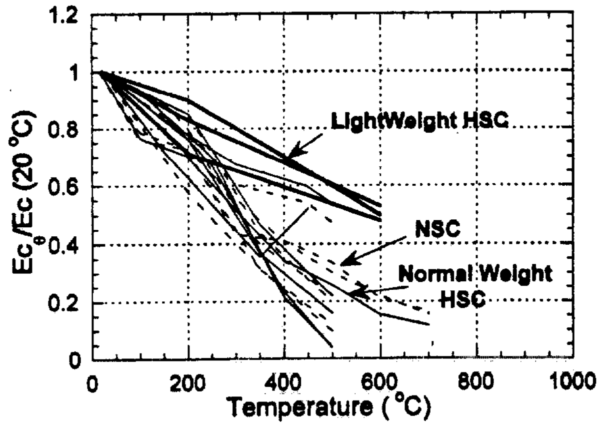


FIG. 5. Modulus of Elasticity–Temperature Relationships for NWA and LWA Concretes from Unstressed Residual-Strength Test Results

TABLE 2. Strength-Reduction Factor $k_{c,\theta}$ for NSC According to CEN (1993, 1994) ENV

Concrete temperature (°C) θ_c (1)	$k_{c,\theta} = f_{c,\theta} / f_{c,20°C}$ NSC Siliceous (2)	NSC Calcareous (3)	LWA (4)
20	1	1	1
100	0.95	0.97	1
200	0.90	0.94	1
300	0.85	0.91	1
400	0.75	0.85	0.88
500	0.60	0.74	0.76
600	0.45	0.60	0.64
700	0.30	0.43	0.52
800	0.15	0.27	0.40
900	0.08	0.15	0.28
1,000	0.04	0.06	0.16
1,100	0.01	0.02	0.04
1,200	0	0	0

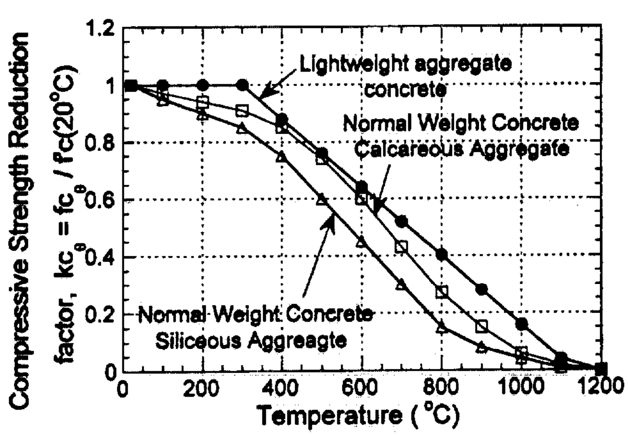


FIG. 6. Strength-Reduction Factor for Concrete with Respect to Temperature (CEN ENV 1993, 1994)

Only a limited number of stressed test data is available. These are compiled in Figs. 3(a and b). The HSC strength–temperature relationships are characterized by the following three stages: (1) Initial strength–loss stage, which begins from room temperature to approximately 100°C for NWA concrete; (2) stabilizing and regaining stage, which begins from 100 to approximately 400°C for NWA concrete; and (3) permanent strength–loss stage, which begins from approximately 400 to 700°C for NWA concrete. Data for LWA HSC under stressed tests are not available.

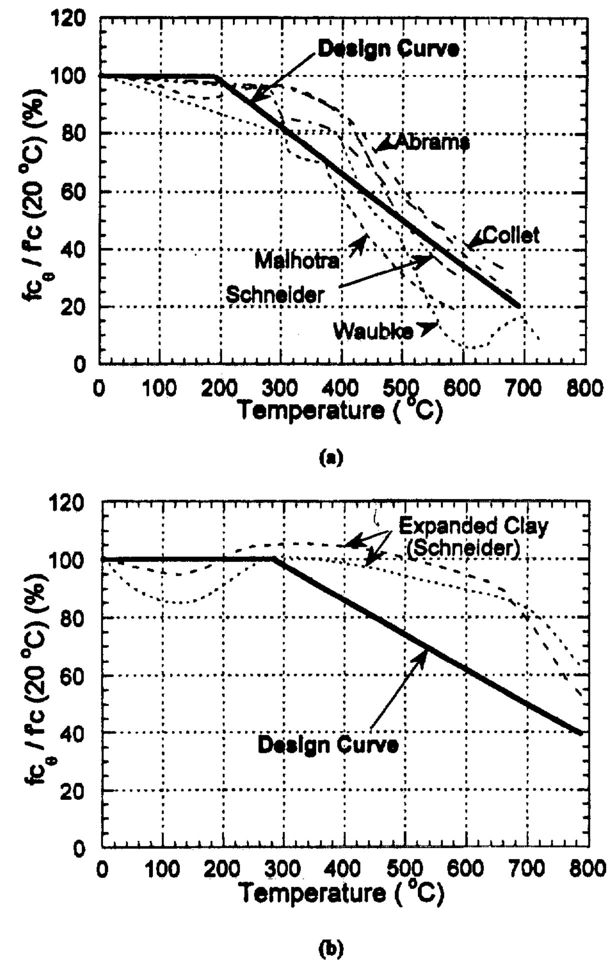


FIG. 7. CEB-Recommended Compressive Strength–Temperature Relationships for: (a) Siliceous NWA Concretes; (b) LWA Concretes

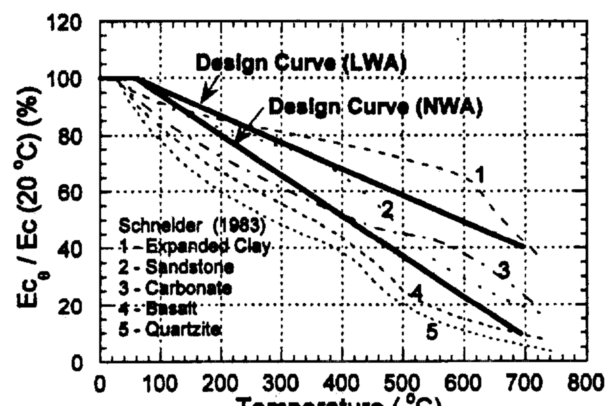


FIG. 8. CEB-Recommended Design Curve for Modulus of Elasticity–Temperature Relationships for NWA and LWA Concretes

Effect of Temperature on Modulus of Elasticity of HSC

For the unstressed tests, there are no significant differences in the modulus of elasticity-temperature relationships for normal-weight HSC (solid, thin lines), NSC (dashed lines), and lightweight HSC (solid, thick lines), as can be seen in Fig. 4. For the unstressed residual–strength tests, the difference in elastic modulus between normal-weight HSC and NSC also is insignificant. However, data for lightweight HSC reveal significantly different profiles of the modulus-temperature relationship versus that of NWA concretes (Fig. 5). These lightweight HSC data were obtained from Hertz's (1984) and (1991) experiments, which used very high strength concrete (specimens with specified strength of 170 MPa). It is not certain to what extent these very high strength concretes influenced this response.

APPLICABILITY OF CURRENT FIRE-DESIGN RECOMMENDATIONS TO HSC

Design Recommendations by Comité Européen de Normalisation ENV (1993, 1994)

Eurocode Nos. 2 and 4 [Comité Européen de Normalisation (CEN) (1993, 1994)] specify rules for strength and deformation properties of uniaxially stressed, NWA (siliceous and calcareous) and LWA concretes at temperatures of up to 1,200°C. In general, compressive strength of concrete at temperature θ (between 20 and 1,200°C) $f_{c,\theta}$ may be obtained by multiplying the corresponding reduction factor $k_{c,\theta}$ by the room-temperature compressive strength $f_{c,20°C}$. Tabulated values of $k_{c,\theta}$ are given in Table 2 and plotted in Fig. 6.

Design Recommendations by CEB (1991)

The CEB Bulletin D'Information No. 208 (CEB 1991) recommended design curves for compressive strength [Figs. 7(a and b)] and modulus of elasticity (Fig. 8) for siliceous NWA concrete and for LWA concrete based on results of tests conducted by Malhotra (1957), Abrams (1971), Waubke (1973), Collet and Tavernier (1976), and Schneider (1983) and by Cruz (1966), Maréchal (1970), and Schneider (1983), respectively. Some of the specimens used in these test programs had a maximum compressive strength of 50 MPa, which exceeds the 40-MPa threshold between HSC and NSC. However, for all practical purposes, the curves in Figs. 7 and 8 are considered to be applicable to NSC.

Applicability of Code Design Recommendations to HSC

An assessment of the applicability of the design curves, prescribed by the Eurocodes (Fig. 6) and recommended by CEB [Figs. 7(a and b) and 8], may be obtained by superposing these design curves onto the results of HSC tests (Figs. 1–5). Figs. 9(a and b) show the superpositions of the design compressive strength–temperature curves on unstressed test data. These figures show that the Eurocode and CEB's design curves are unconservative in predicting the compressive strength of normal-weight HSC in the temperature range between room tem-

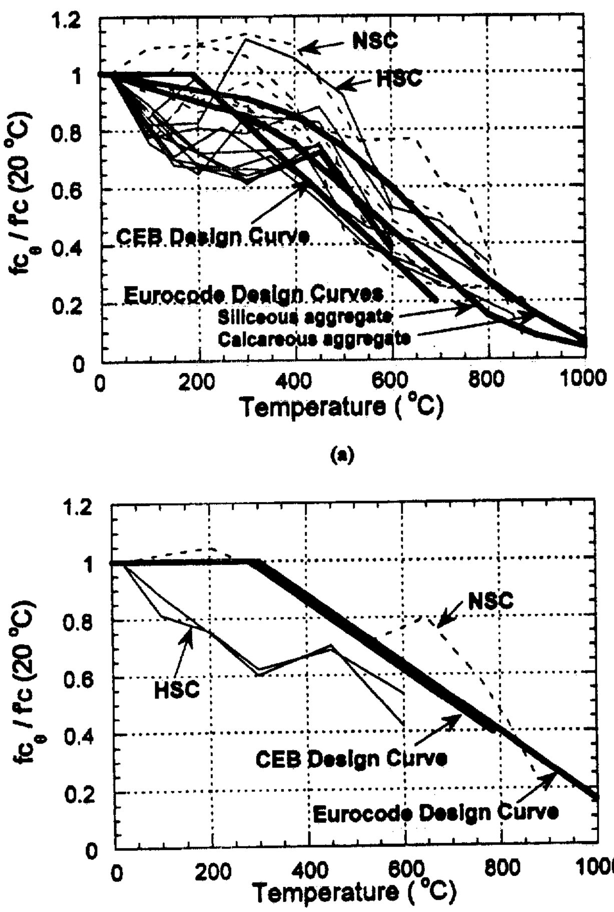


FIG. 9. Comparison of Recommended Design Curves for Compressive Strength of: (a) NWA Concretes; (b) LWA Concretes and Results of Unstressed Tests

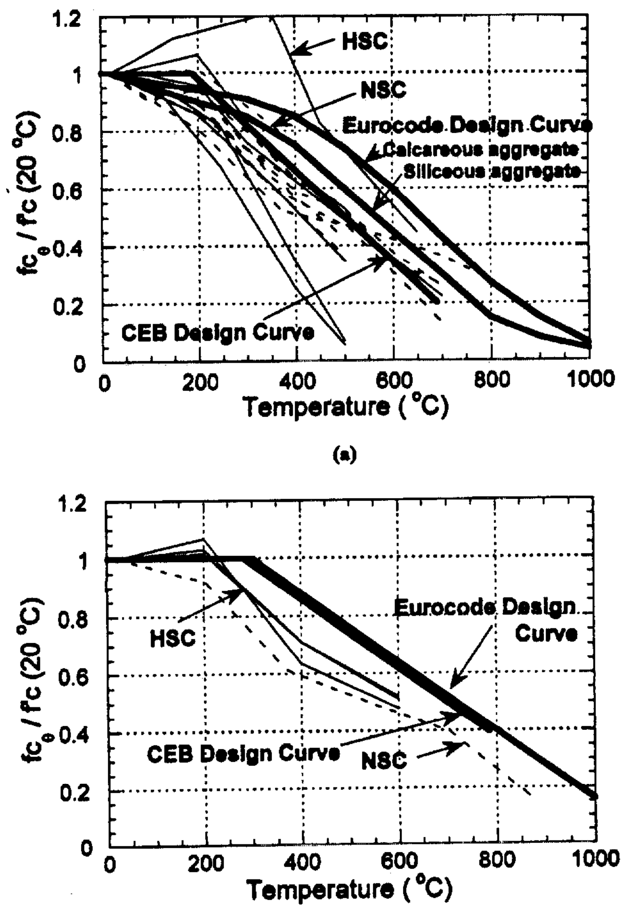
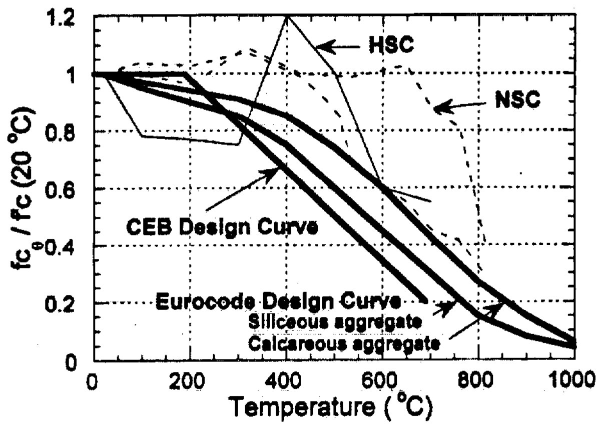


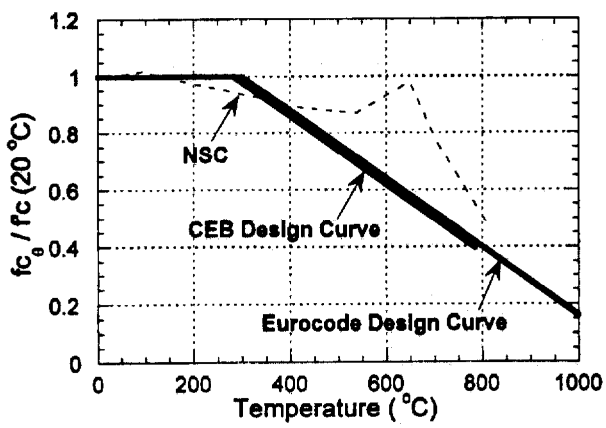
FIG. 10. Comparison of Recommended Design Curves for Compressive Strength of: (a) NWA Concretes; (b) LWA Concretes and Results of Unstressed Residual–Strength Tests

perature to approximately 350°C. Above 350°C, the design curves become more applicable to both HSC and NSC, which is consistent with reported experimental observations that the difference between HSC and NSC narrowed at this temperature. For LWA concrete, Fig. 9(b) shows that, with the limited amount of data available, both the Eurocode and CEB's design curves are unconservative for HSC and are more suitable for NSC.

Figs. 10(a and b) show the superpositions of the same design curves on unstressed residual-strength test data. These figures show that the Eurocode and CEB's design curves are



(a)



(b)

FIG. 11. Comparison of Recommended Design Curves for Compressive Strength of: (a) NWA Concretes; (b) LWA Concretes and Results of Stressed Tests

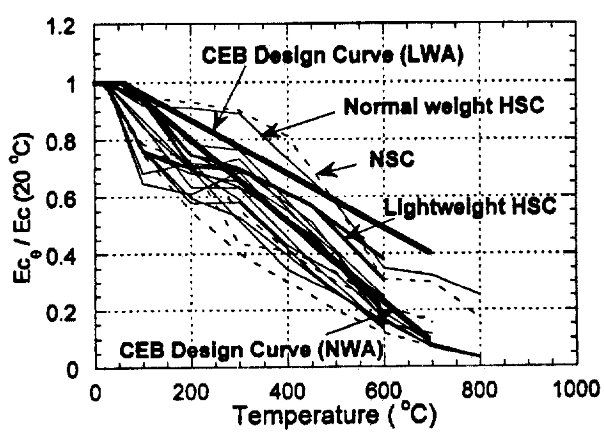


FIG. 12. Comparison of Design Curves for Modulus of Elasticity and Results of Unstressed Tests

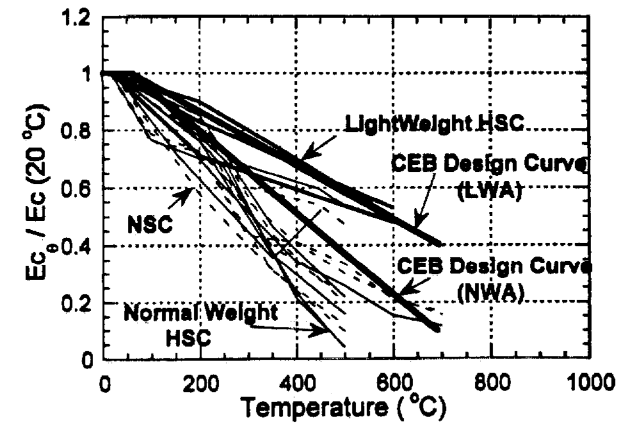


FIG. 13. Comparison of Design Curves for Modulus of Elasticity and Results of Unstressed Residual-Strength Tests

in better agreement with HSC data than for unstressed tests. However, the design curves appear to be slightly unconservative for both HSC and NSC at temperatures above 250°C. For LWA concretes, the design curves also are unconservative for both HSC and NSC.

Similarly, Figs. 11(a and b) show the superpositions of the design curves on stressed test data. Given the limited amount of stressed test data available, it is difficult to arrive at definitive conclusions regarding the applicability of the design curves. In general, the design curves do not appear to be especially representative of either HSC or NSC. However, of these they do tend to exhibit closer conformance to NSC.

For modulus of elasticity, Fig. 12 shows that the CEB design curves are unconservative compared with unstressed test data for both NWA and LWA HSC. For the unstressed residual-strength test, the design curves for LWA appear to be in good agreement with data from LWA HSC. However, the design curves for NWA concrete remain unconservative compared with data of NWA HSC.

From these superpositions, it may be concluded that the current design compressive strength and modulus of elasticity-temperature curves prescribed by the Eurocodes (CEN 1993, 1994) and recommended by CEB (1991) are more relevant to NSC than to HSC.

SUMMARY AND ANALYSIS

In this paper, results of fire tests of HSC and NSC were compiled and compared for differences in mechanical properties of concretes at elevated temperature. The compiled test data also were compared with design provisions prescribed by the CEN Eurocodes (CEN 1993, 1994) and recommended by CEB Bulletin D'Information No. 208 (CEB 1991) to examine the applicability of these design recommendations to HSC. Important trends concerning the performance of HSC at elevated temperature are revealed from the comparisons. The following summarizes the key findings obtained from these comparisons:

1. The material properties of HSC vary with temperature differently than those of NSC. The differences are more pronounced in the temperature range between 25 and approximately 400°C, where higher-strength concretes have higher rates of strength loss than lower-strength concretes. These differences become less significant at temperatures above 400°C. Compressive strengths of HSC at 800°C decrease to approximately 30% of the original room-temperature strengths.
2. For unstressed and stressed tests of HSC, the variations of compressive strength with temperature are characterized by the following three stages: (1) An initial stage of

strength loss (25 to approximately 100°C), followed by; (2) a stage of stabilized strength and recovery (100 to approximately 400°C); and (3) a stage above 400°C characterized by a monotonic decrease in strength with increase in temperature.

3. For unstressed residual–strength tests of HSC, the compressive strength versus temperature relationships are characterized by the following two stages: (1) An initial stage of minor strength gain or loss (25–200°C) followed by; (2) a stage above 200°C in which the strength decreases with increasing temperature.
4. The strength recovery stage of higher-strength concretes occurs at higher temperatures than lower-strength concretes.
5. A temperature of 300°C marks the beginning of higher rate of decrease in modulus of elasticity for all concretes. The LWA concretes retain higher proportions of the original modulus of elasticity at high temperature than NWA concretes. The difference is more pronounced for unstressed residual–strength tests than for unstressed tests.
6. Current design recommendations for compressive strength and modulus of elasticity of fire-exposed concretes are more relevant to NSC than HSC. The Eurocode and CEB design curves have questionable application to HSC, if any at all.
7. Experimental results support that HSC is more susceptible to suffer explosive spalling failure when exposed to high temperature (above 300°C) than NSC. Additional research and testing is needed to verify this from a theoretical standpoint.

Given the increased usage of HSC in structural applications, the behavioral differences observed for HSC and the inapplicability of existing code provisions for fire-exposed HSC must be recognized and addressed so as to reduce the likelihood of premature structural collapse in the event of fire. The amount of test data on fire-exposed HSC available to date is insufficient relative to the number of variables (concrete strength, aggregate types, test conditions, specimen size, concrete density, concrete permeability, concrete porosity, heating rate, etc.). Of particular interest are data from the stressed tests, which simulate the condition of structural elements when exposed to a fire. Such data are scarce, as can be seen from Figs. 3(a and b). Data also are scarce for fire-exposed HSC made of LWA concretes and tested under all three types of tests [see Figs. 1(b), 2(b), and 3(b)]. Furthermore, to effectively address the behavioral differences observed in HSC, especially explosive spalling, the effects of a number of variables such as those listed earlier must be quantified. Also, the variation of the stress-strain relationships of HSC with temperatures must be established experimentally. Such relationships are not reported widely in the existing literature but are essential for the development of constitutive models of HSC to predict structural performance during a fire.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

Ec = elastic modulus of concrete at room temperature (20°C);
Ec_θ = elastic modulus of concrete at temperature θ;
fc_θ = concrete compressive strength at temperature θ;
f'_c = concrete compressive strength at room temperature (20°C); and
kc_θ = compressive strength–reduction factor [fc_θ/f'_c (20°C)].